Exhibit A

Revision Issue: 1 March 2, 2021

Preamble: This document has been drafted during a period of discovery of facts related to an existing drainage system installed in the Tidalholm Subdivision. Some of the key assumptions incorporated in this document are still being researched. This document must be considered a **Preliminary Draft**

Stormwater Flow Characteristics of a 36 " Reinforced Concrete Pipe Tidalholm Village Diverter Line

Introduction

Tidalholm Village is a 137 Lot Single Family subdivision in the Federal Point Township, New Hanover Cty., NC. It is directly adjoined by the Monterrey Heights Subdivision. The original stormwater design for Tidalholm Village received approval from the NC Dept of Environment, Health, & Natural Resources on 12/19/1994. The original design included a collection system [intakes & underground RC Pipe], 11 vegetative swales, and a detention pond. In addition to the total Tidalholm Village project, the design was to handle runoff from an offsite area of 600,000 sq ft.

After the system and area, specifically the intersection of Lipscomb Dr. and Point Reyes Dr experienced significant flooding, the stormwater management design was re-evaluated by NCDENR, and was designated to be inadequate in a "Notice of Compliance Violation" letter issued on 4/18/2000. The compliance violation related to an internal drainage basin area of about 9 – 13 acres that was serviced by a detention pond located between lots 129 & 130.

- Drainage from about 75% of Monterrey Heights subdivision is entering Tidalholm Village via Point Reyes Dr.: drainage area (*offsite*) is in excess of the 600,000 sq ft allowed in the design calculations (*for detention pond between lots 129 & 130*).
- This deficiency must be corrected by either routing the stormwater runoff around the detention pond, or accounting for the increased volume in the detention pond design.

A stormwater management investigation and onsite survey of the Monterrey Heights subdivision determined the actual offsite drainage to be ca. 3,700,000 sq ft. This result defined the (*over permit*) area to be: 3,700,000 sq ft (-) 600,000 sq ft (=) 3,100,000 sq ft; or 71.17 acres.

The subdivision developer proposed the install of a weir box diverter in the main 36" underground stormwater pipe upstream of the detention pond. Conceptually, this diverter replaced a portion of the 36" RCP [Reinforced Concrete Pipe] with a large box, with an internal weir set at the maximum normal water level of the detention pond [~15.15 ft]. Increased levels in the detention pond and/or high flows through the 36" pipe would lead to an overflow condition at the weir box. Stormwater that overflowed the weir would be routed through a

new 36” RCP along a drainage easement between lots 127 & 128. The new RCP then would continue into an off-property drainage area until it was to discharge at a point where the RCP invert matched the ground elevation [11.9 ft +/-]. The ultimate destination of the diverted flow is a wetland at an elevation of less than 9 ft, located about 600 ft overall distance to the west from Lipscomb Dr.

Tripp Engineering, P. C. submitted design drawings of this proposal, for review by NCDOT Hydraulics Div, on Dec 17, 2001. Handwritten notes on discovery documents suggest this design review was not completed until 2003.The final diverter system design was presumably installed in 2003 or later.

There are several discrepancies in Diverter System design found among the many documents discovered to date. I have analyzed the actual configuration of the Diverter System and used that information to determine the capacity of the piping system. The “As Built” length of the final segment of RCP, extending beyond lot 128, is only ~33 ft, thus the estimated overall length of the new 36” RCP is 191.5 ft, and includes two bends to facilitate a ~37 ft long offset to the south. The discharge element is a “flared end section” [see Fig 5] which widens the flow area at the discharge point, thereby reducing erosion and the resistance to flow. The discharge location information is found on an adjoining subdivision [Tralee Ct] drawing [Dwg. C1], which identifies the discharge elevation as 12.36’. A properly functioning ditch is required to avoid obstructing the discharge water flow from the outfall location. A survey of the ditch area topography was completed prior to the install, but no survey data was found to completely describe the post install geometry.

<u>Description of Study Cases</u>

The analysis of the “As Built” installed 36” diameter Reinforced Concrete Pipe [RCP] flow capacity requires a definition of the discharge geometry of the “ditch” receiving the flow. The standard “ditch” implied and used for the purposes of this study was a uniform trapezoidal ditch. The flat bottom of the ditch, for calculation purposes, was assumed to be 4 ft width and the sides slope at a 1/1 pitch. The liquid flow rate versus depth of water for the ditch was determined using an on-line model which calculates results from the “Manning equation”. A sensitivity check analysis was performed for an alternate ditch of 3 ft width with 2/1 side slope. Results indicated that design would operate at 2-3 inches lower levels at the same flows.

The level in the ditch for any given flow was then used to determine a head loss associated with the change in cross section from pipe discharge, to flared expansion section to ditch. The ditch level and transition/expansion losses are summed and included with the overall pipe losses. The result is a required elevation difference [head required, in ft of fluid] between the RCP discharge invert, and the inlet of the RCP inside the diverter box. The slope of the (invert) of the ditch was assumed to be 0.4 ft/ 100 ft. This slope delivers flow to the approximate elevation of the planned discharge point.

There were four separate study cases analyzed, which represent different conditions at the discharge of the pipe into the ditch; plus a separate special “Partial RCP” Case.

CASE 1: The RCP discharge is above the level of water in the ditch; *this case, although feasible, Was probably not done because of the increased elevation difference needed, longer ditching length, and erosion control requirements.*

CASE 2: The RCP discharge invert is at the same level as the ditch inlet invert; *Base Case; this is identified as a normally operating system, without the major obstructions introduced in the following cases.*

CASE 3: Same as Case 2, but the inlet to the ditch is obstructed; *obstruct dam height is 2 ft.*

CASE 4: Same as Case 3, but an 18 inch depth obstruction extends inside the RCP; this represents *choked flow because the obstruction reduces the open cross section for fluid flow to 50%.This case is thought to be consistent with the upward spouting of discharging water reported by observers during the storm*

CASE 4A: Same as Case 4, but the 18” depth obstruction [50% cross-section area] inside pipe is increased to 24” to yield 70% obstruction of the pipe cross-section.

Partial RCP: Flow vs. head was calculated for the pipe from the first 45*L* bend to the discharge

<u>Summary of Results</u>

The results; shown in Figure 2, are all based on a trapezoidal ditch; 4 ft width at the bottom and 1/1 side bank slopes, invert slope of 0.4%, [a ~1.0 ft drop over the ~250 ft length of a proper ditch required to deliver water to a flat plane area]. A larger ditch and/or greater slope would have improved the performance of the system. Figure 1 displays the elevation profiles of the natural surface, the RCP piping and the needed open ditch invert elevations for the “As Built” study cases. We assume Case 1 ditching requirements were not provided.

<u>The best analysis of a normally functioning ditch/36” RCP configuration therefore is shown by Case 2 results</u>. In Study Case 2 the system is capable of delivering about 30,000+ gpm of stormwater, with no water buildup on the street, at a total head required of <5 ft [RCP inlet to RCP invert]. The flow capability would increase to ~35,000 gpm for moderate [~ 14 inch] water buildup on the streets. If the actual flood level reached ~20 ft elevation [~3+ ft buildup], as described by residents, the “clean RCP system” would have delivered a flow rate of ~41,000 gpm to the outfall area.

The consequences of obstructions in the ditch and pipe were evaluated in Study Cases 3, 4 & 4A. In Case 3 the ditch was assumed partially blocked, near the RCP discharge, by a 2 ft

high (above ditch invert) berm. The results show that generally the height of the obstruction was translated to an increase elevation of water at the RCP inlet. For low flows the translation is 1/1. For higher flows the ditch width and cross section increase diminishes the effect of the obstruction. <u>For example; the increased level of water for 30,000 gpm flow was 1.3 ft because of the added 2 ft high obstruction</u>.

In Case 4 the obstruction of flow was compounded by adding what is equivalent to a sediment buildup that is 18 inches in depth and extends back inside the circular cross section of the RCP. This is what I called a "choked flow" element because this restriction acts independently by reducing the effective cross section of the 36"ID pipe. This restriction and accompanying pressure drop varies with the velocity heads occurring in the remaining open, unrestricted section of pipe; it increases as a function of velocity squared. <u>For example; the increased level of water for 30,000 gpm flow was 3.7 ft because of the added 18 inch layer of sediment inside the pipe</u>. Case 4 also includes the Case 3 partial ditch blockage. <u>So, for example, the overall water elevation increase vs. the "normal" Case 2 level is 4.9 ft. for 30,000 gpm flow</u>. The Study Case 4 [obstructed flow] of ~30,000 gpm would have caused the water level to increase to the ~20 ft elevation observed. Several "after Florence flooding" pictures of the discharge area indicate that the obstruction was likely substantially greater than the 2 ft height studied in Cases 3 & 4. On scene reports by residents indicate that the discharge was very obstructed and the water discharging from the RCP was "spouting upward". That "on scene" report tends to confirm the presence of an obstruction which extends into the mouth of the discharge piece.

Case 4A was added to show the results of increasing the restriction inside the pipe. When the open area is reduced to 30% [70% restriction] the flow capacity of the pipe again reduces dramatically. Water would have to appear on the street before the diverter could deliver 12,000 gpm. At the flood levels reported [20 ft elev], the diverter line would have only delivered a very modest 17,000 gpm; about 40% of the clean system capacity.

The combination of calculated results and storm observations indicated that just the two defects in the study cases, combined, could account for the substantial flooding that was observed during Hurricane Florence.

FIG 1 - SECTION VIEW OF DIVERTER SYSTEM

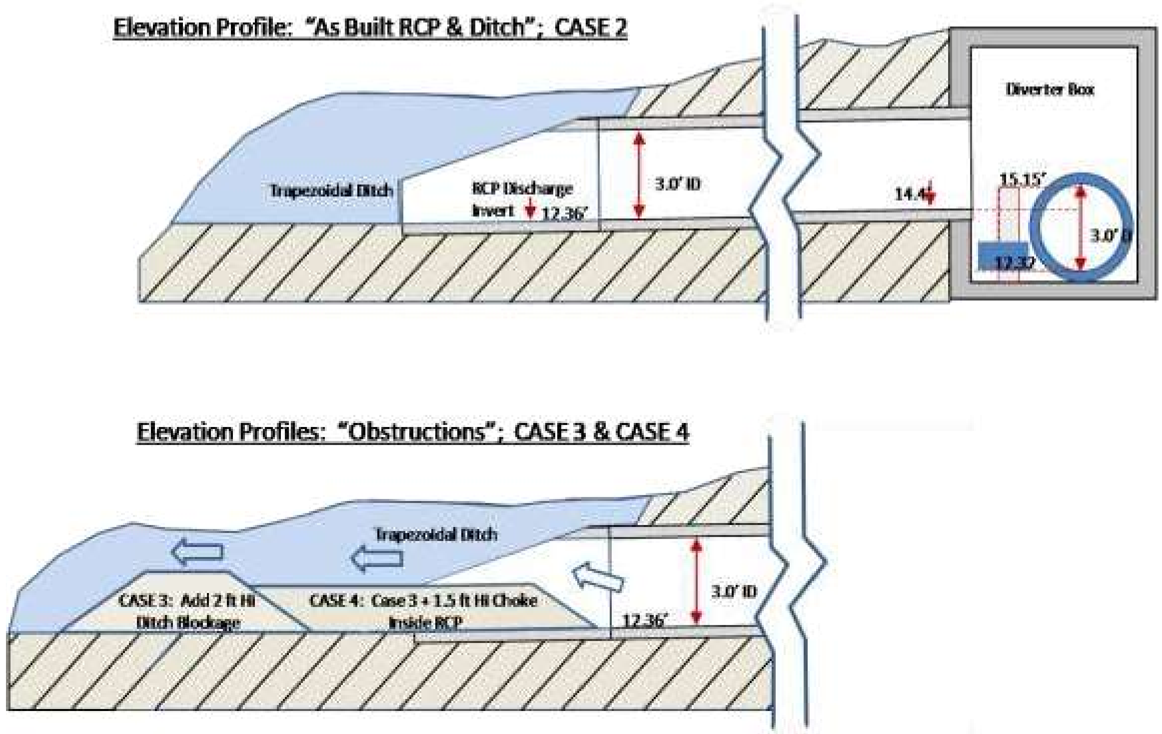


FIG 2 - GRAPH OF STUDY CASE RESULTS

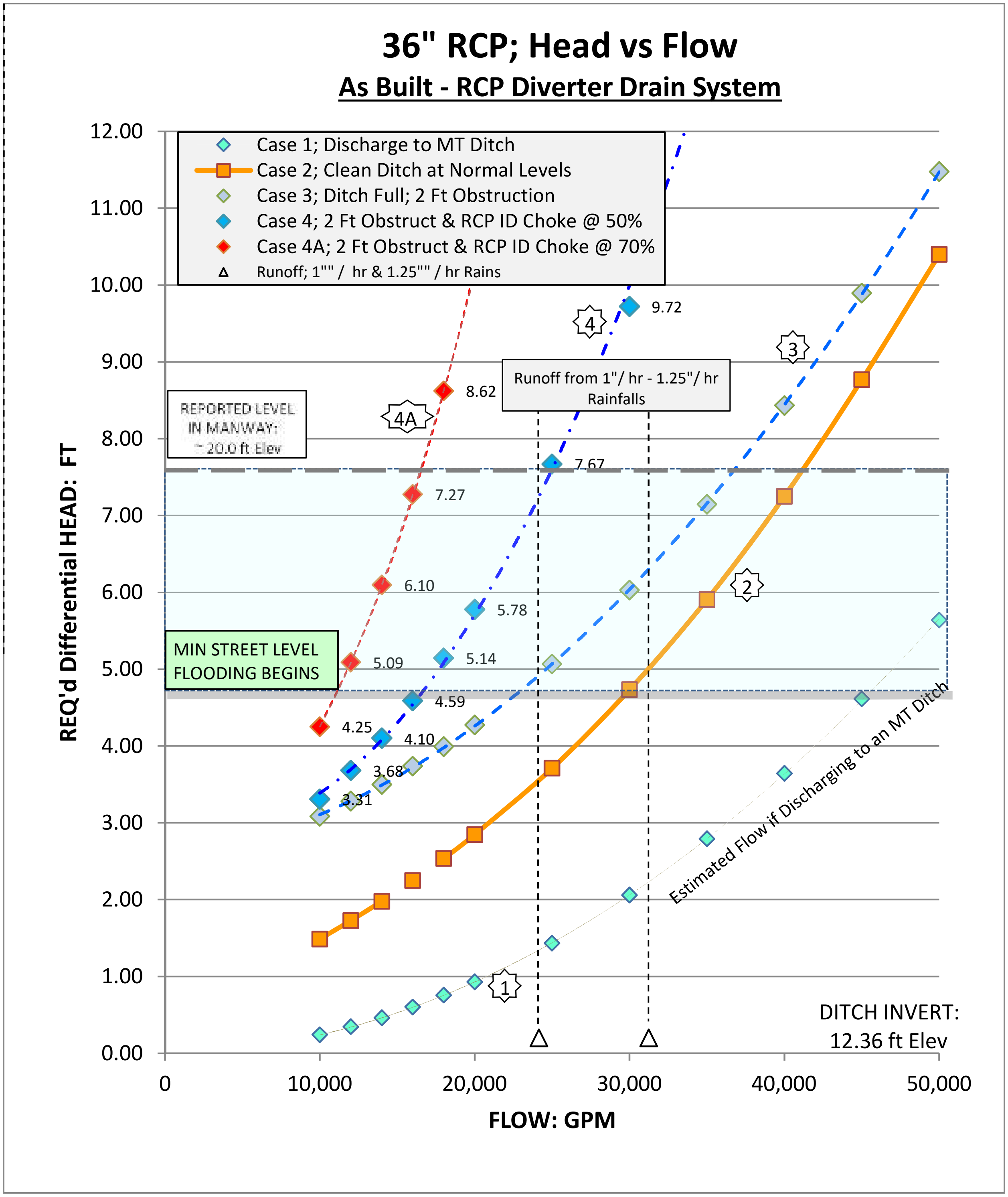


FIG 3; Table of Study Case Results; Case 2 [Normal - Clean Ditch]

Flow; GPM [gal/min]	Flow; CFS	Velocity Heads	Ditch Water Depth [ft of water]	TDH of Pipe [ft of water]	Elev at Inlet [ft above sea level]
10,000	22.28	0.154	1.24	0.25	13.85
12,000	26.73	0.222	1.38	0.35	14.09
14,000	31.19	0.303	1.50	0.50	14.34
16,000	35.64	0.395	1.62	0.63	14.61
18,000	40.10	0.500	1.73	0.80	14.89
20,000	44.55	0.617	1.84	1.02	15.21
25,000	55.69	0.965	2.07	1.64	16.07
Water Backs Up Onto Street [~17 ft Elev]					
30,000	66.83	1.389	2.29	2.44	17.09
35,000	77.97	1.891	2.48	3.43	18.27
40,000	89.10	2.470	2.67	4.58	19.61
45,000	100.24	3.126	2.84	5.93	21.13
50,000	111.38	3.859	3.00	7.40	22.76

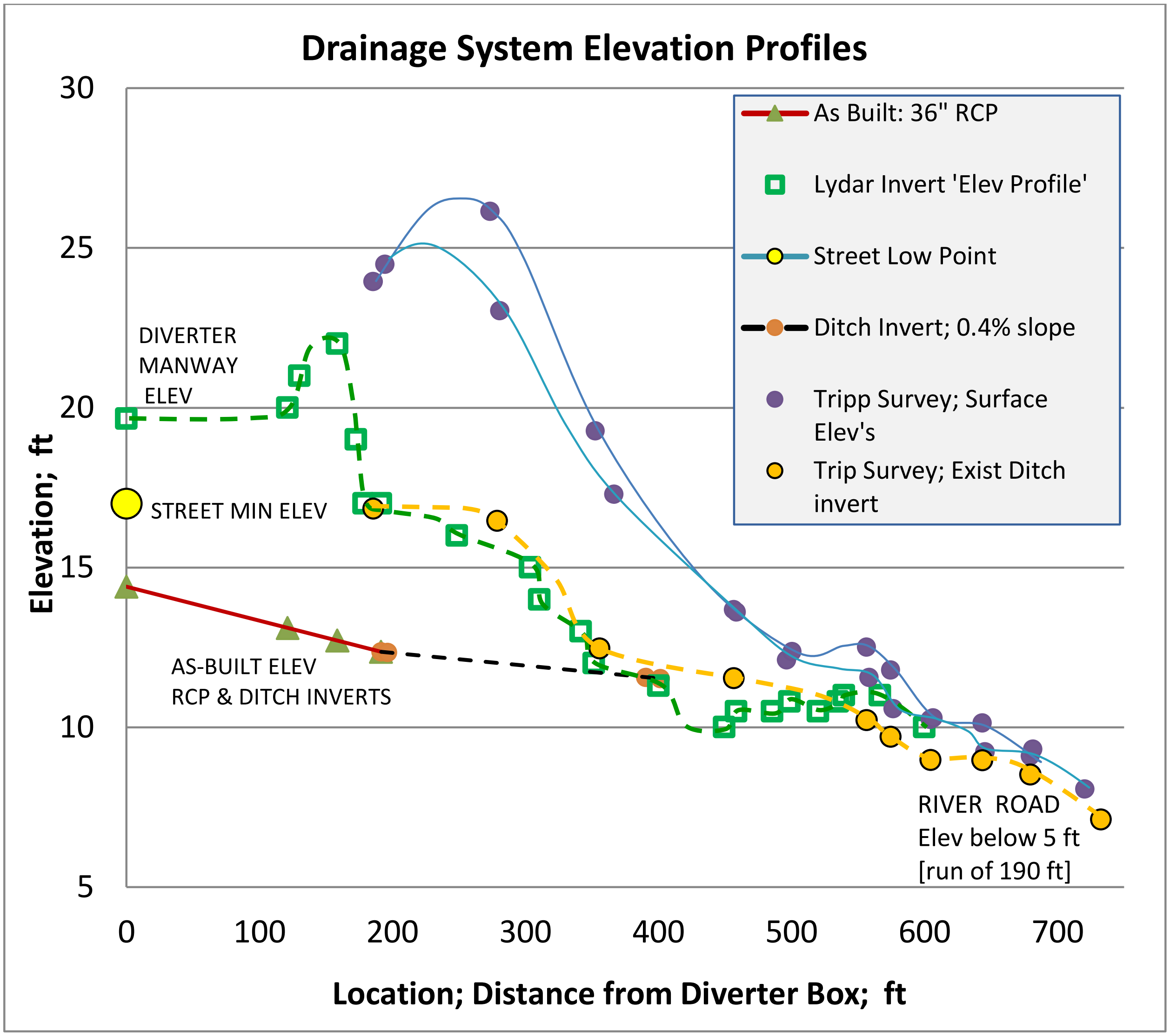
Note: [Normal – Clean Ditch] is ; COE roughness coefficient (n) = 0.025 "earth channels, in fair condition, some growth". Roughness coefficient (n) = 0.035 "natural streams and canals in poor condition, considerable growth" would increase level less than 5.4 inches at 30,000 gpm.

ADDENDUM: FURTHER CALCULATION BASIS DETAILS

Pipe – Discharge Point Ditch Configuration Requirements

The discharge flow of stormwater from the RCP requires some form of flow path to arrive at low lying areas which adjoin the Cape Fear River. The destination area lies on the east side of River Road, including relatively flat areas, at an elevation of < 11 ft; and only ~300 feet to the west of the RCP discharge point. It is assumed that the flow at this point would have widened out and the surface elevation requirement for the flow would be decreased. The intermediate elevations, as shown on NHC Lydar Topo Maps, require ditching to provide a proper, and non-restrictive path for water flow. The current version of the NHC Topo Map [formulated from a 2002 DTM] does not indicate that this ditch existed to the depth required. Tripp Eng. P C, provided a field survey, transmitted to NCDOT 1-21-2002, of ditch elevations. This map, with survey elevation mark-ups, is thought to be pre-installation . Locations closer to the Tidalholm property line are all above 16.4 ft, so significant excavation would have been required to provide a proper ditch for the "As Built" length pipe.

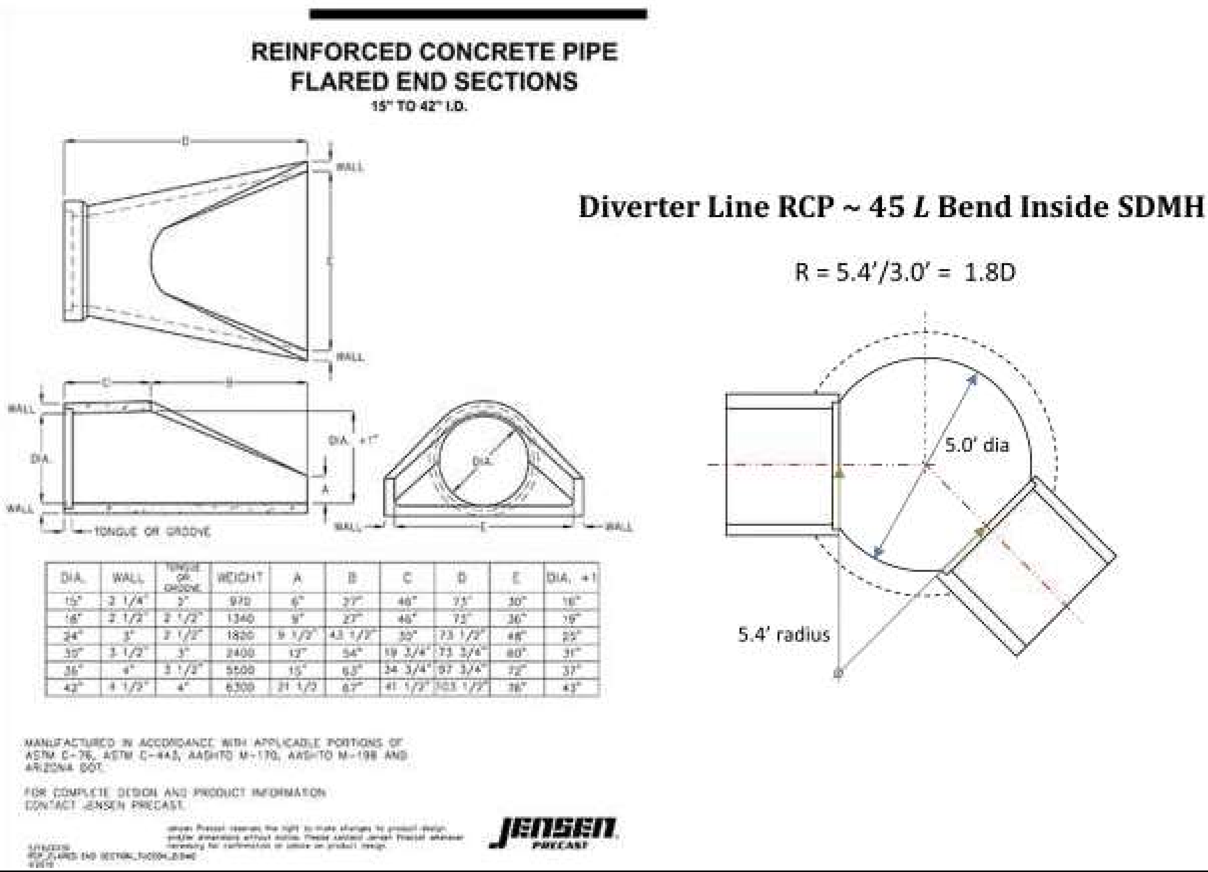
FIG 4 - DRAINAGE SYSTEM ELEVATION PROFILES



In order to evaluate the pipe flow we must understand the configuration of the pipe to ditch flow feature. Additionally, the ditch cross section geometry, surface roughness, and ditch slope determine the operating level of the ditch at any given flow. I used the cross section of a trapezoidal ditch, 4 ft width at the bottom, with 1/1 slope for the sides. The surface roughness factor used was, as presented in tables from several sources, for a “good condition” ditch. The entrance elevation of the ditch was assumed to have exactly matched the “invert” elevation of the RCP Expansion Discharge piece (12.36 ft). I used the “Manning’s Formula” online flow

calculator available at Eng.Auburn.Edu, for open channel drains of various cross sections. In all cases a ditch slope [0.4%] and the ditch level were input variables and the flow rate was the output variable. Multiple results were cross checked using other available online software; crosschecked results agreed within 0.3%.

FIG 5 - Drawings of Flared End Section & RCP "Bends"



DIA.	WALL	TONGUE OR GROOVE	WEIGHT	A	B	C	D	E	DIA. +1
15"	2 1/4"	3"	970	6"	27"	46"	73"	30"	16"
18"	2 1/2"	2 1/2"	1340	9"	27"	46"	73"	36"	19"
24"	3"	2 1/2"	1820	9 1/2"	43 1/2"	30"	73 1/2"	48"	25"
30"	3 1/2"	3"	2400	12"	54"	19 3/4"	73 3/4"	60"	31"
36"	4"	3 1/2"	5500	15"	63"	34 3/4"	97 3/4"	72"	37"
42"	4 1/2"	4"	6300	21 1/2	87"	41 1/2"	103 1/2"	78"	43"

<u>Reinforced Concrete Pipe Flow Calculations</u>

For the purposes of flow and pressure drop [differential head] calculation, the pipe is divided into runs of strait pipe and included features; 1: inlet contraction [from inside box to inside pipe], 2: cumulative straight pipe runs, 3: ~ 45< bends in pipe (2), and 4: the discharge expansion to the ditch [includes a "flared end section"]. Since the flared end section has a cross-section two times the width of the pipe ID and naturally morphs into the trapezoidal ditch cross-section; the inlet of the flared end section to ditch cross-section are treated as one feature.

1. The inlet to the 36" pipe is located at the inside edge of the wall of the weir box. Since the RCP has a slip joint "rabbit" of 2" increased radius, which is included in a smooth mortar joint, the effective r/D radius of the "sharp edge contraction" is 2"/36" or 0.055. Note that field inspection revealed even longer radii because the RCP leading edge was not flush with the inside wall and the void had been filled with mortar. The effective r/D of the "sharp edge contraction" used was " = 0.055. The effective K used was 0.158. Interpolated from "Crane; Flow of Fluids, Tech Paper #410, p A-29, Fig for "Pipe Entrance". (K factors for losses):K = 0.15 for r/D = 0.06. K = 0.04; for r/D at 0.15 or greater. This loss coefficient is multiplied times velocity heads to obtain the "head loss" for the feature.

2. The two bends in the pipe (~45°*L*) are actually executed in ~5-6 ft diameter manhole access wells. The bend friction loss net effect is better modeled as a short radius elbow, as opposed to a sharp miter bend. The effective bend radius is ~ (1.8) pipe diameters. The friction loss of each bend is then converted to an equivalent strait pipe length. The bend friction loss was estimated from Crane; A-27, Resistance of Miter Bends; also see E. I. DuPont Co. Engineering Standard [DG 5B]; Fig 2 (p 11) relating bend loss for various configurations in the number of pipe diameters that may then be added. In our case, determined from the equation:
   L/D = 11.38 and: Ft Equiv = (11.38 x 3.0 dia.) = ~ 34.1 ft

3. The strait pipe is composed of three segments: total length is: 121.2+37.3+33 = 191.5 ft. The two ~45L bends are included as 5 ft physical dimension, but 34.2 ft equivalent length each. Total equiv length straight pipe of bends = 58.3 ft. Total equivalent pipe length = 249.8 ft.
   The head loss in the total aggregate equivalent pipe length was determined for all the flow rates studied by using head loss tables presented in "Cameron Hydraulic Tables; p 3-29". The pipe surface roughness was corrected for increased roughness of concrete versus the cast iron in the table by adding 20% to all the head loss values. The correction factor was based on an analysis of the Fanning friction factors (f_t); as derived from a Moody Diagram using a surface roughness (ε) of 0.0004 for asphalt coated cast iron, and 0.001 for smooth concrete.
   The Reynolds Number was determined from the following equation: $N_{RE} = D\, v\, p / u_e$ [Crane Flow of Fluids; Tech paper #410: p 1-4]. For 10 ft/sec velocity,
   ~ 31,720 gpm $N_{RE} = (3.0 * 10 * 62.4) / 0.000673 = 2.78 * 10^7$
   The Fanning friction factor for the pipe is then determined from a graph of friction factors vs. Reynolds Numbers [Crane Flow of Fluids: p A-24]

f_t = 0.0125 "for surface relative roughness of 0.0013 [asphalt coated cast iron]"

f_t = 0.015 "for surface relative roughness of 0.00033 [smooth concrete]"

Alternatively, Cameron Hydraulic Data; for flows between 30,000 & 45,000 gpm uses a Fanning friction factor of; f_t = 0.0134 for [asphalt dipped cast iron]. Since I used their tabular data, the implied Fanning friction factor for the +20% adjustment to RCP is; f_t = 0.0161

4. The (maximum) resistance to flow due to a sudden enlargement is expressed by $K = [1 – (d1^2 / d2^2)]$ therefore equally = $[1 – (A1 / A2)]$; Crane, Equation 2-9. If the enlargement occurs at an included angle less than 45°*L*, the loss may be reduced by the correction for the included angle θ: $Ce = 2.6 * \sin (\theta/2)$; Crane, Equation 2-12. I used the more conservative full expansion loss in the report. However, the Flared End Section would loosely correspond to a Ce of < 0.5 for a portion of the expansion. That implies the calculations are conservatively high with respect to the expansion losses into the ditch cross section.

5. The total system loss was tabulated by adding the discharge transition loss and the entrance loss to the equivalent pipe length loss. Determining the RCP inlet water elevation, above the RCP discharge invert, requires adding the water level in the ditch. The reference total head (elevation in feet of water), in all cases, are from the invert elevation of the RCP discharge. These calculations assumed that the RCP was running full up to the flared end section. At very low flows, which are not of primary interest for this study, this wouldn't be true. In any case that this deviation occurred, the results given would be conservative with respect to differential head. See Figure 2 and Table 3 for all case results. Note that CASE 1 results assume that the ditch operating level is in all cases lower than the RCP discharge invert.

<u>Ditch Flow Sensitivity to Condition</u>

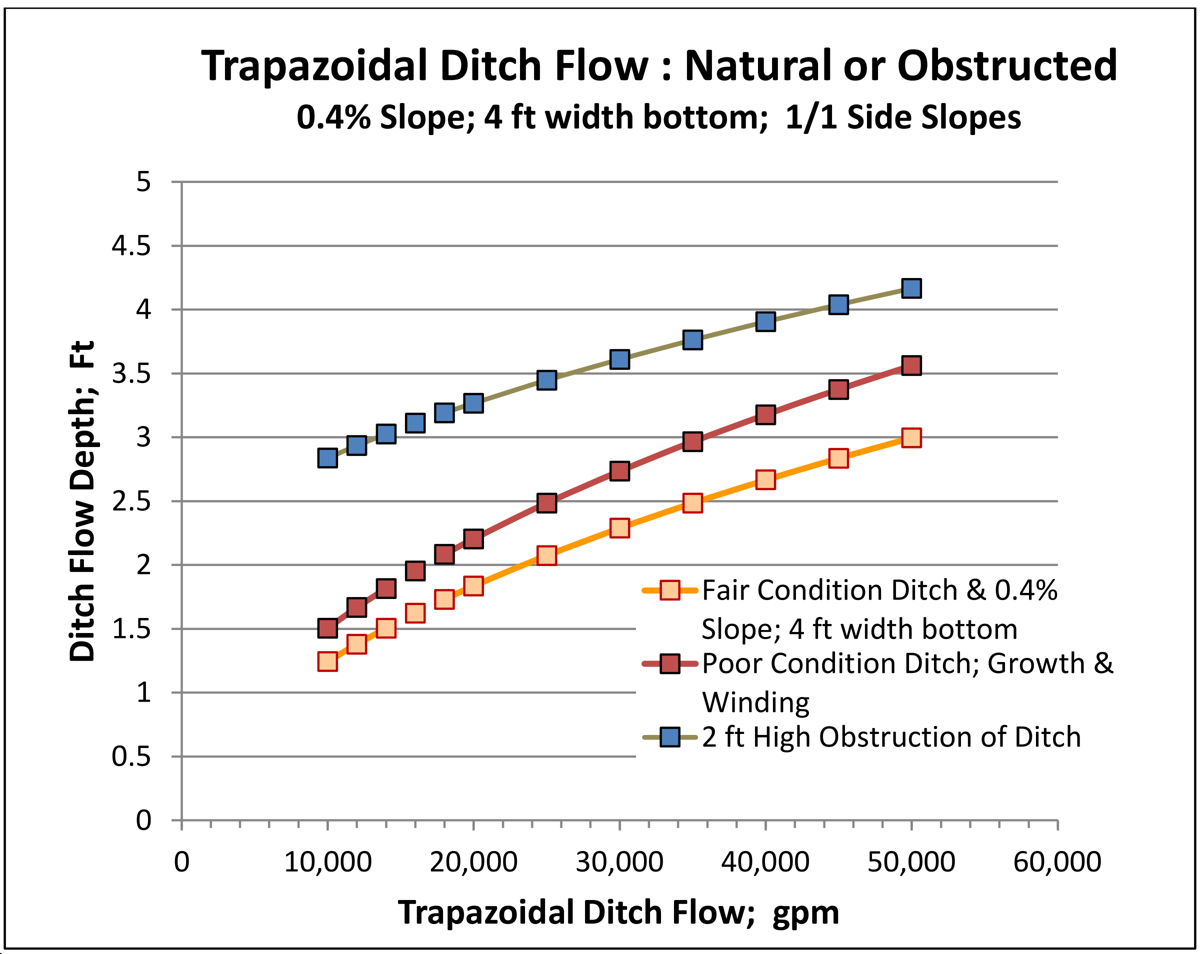
An analysis was done to determine how flow capability of a trapezoidal ditch is affected by the general condition of the ditch. The analysis was based on the Manning equation for ditch flow as a function of the "roughness factor" (n). The U S Army Corps of Engineers Engineering Manual; EM1601, Chap2 Open Channel Hydraulic Theory presents the Manning roughness coefficients for different boundary types in Table 6.1

- [n = 0.017] = Earth Channels in Best Condition
- [n = 0.020] = Straight Earth Canals in Good Condition

- [n = 0.025] = Rivers & Earth Canals in Fair Condition; some Growth
- [n = 0.035] = Winding Natural Streams & Canals in Poor Condition; considerable Growth
- [n = 0.041-0.050] = Streams with Rocky Beds/Variable Sec Rivers; Some Bank Vegetation

A coefficient of 0.025 was used in the "Base Case" [Case 2] for the differential analysis. This was assumed to represent the ditch when either relatively new or "maintained" to represent performance for "Fair Condition; some Growth". The exact same ditch was evaluated for a surface roughness of 0.035; to represent a ditch which had not been maintained and had deteriorated to "Poor Condition; Considerable Growth". The results are tabulated in Figure 6 below. The ditch in "Poor Condition" [n = 0.035] increased the operating level by about one half foot at 40,000 gpm flow. This was still considerably better than Case 3 where the ditch has an obstruction consisting of 2 feet of sediment (see Fig 6 following).

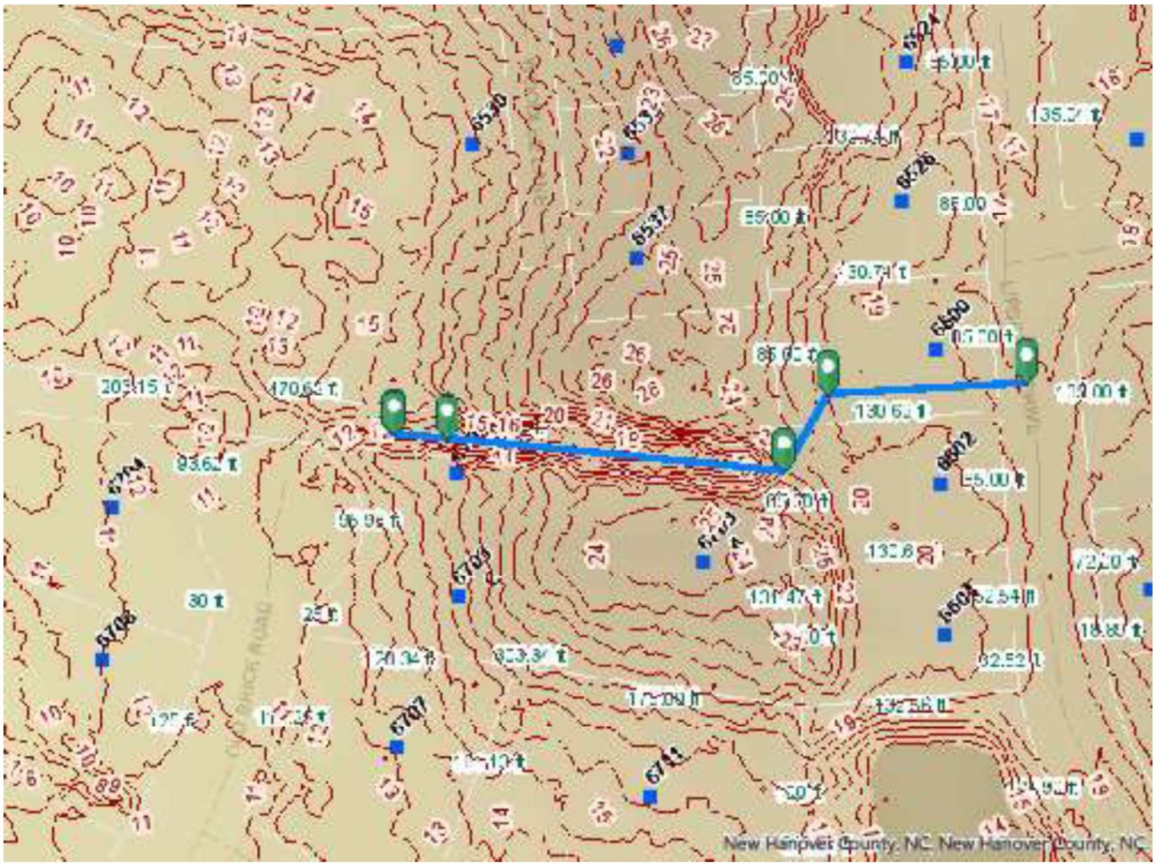
FIGURE 6:   TRAPEZOIDAL DITCH FLOW vs. DEPTH



Other Miscellaneous Background Data & Observations

1. The residents' (lot 127) observation (during the storm and flooding) of the level in the first bend manhole is very instructive. The elevation at that point in his yard is between 20 – 21 ft. The resident reported that the manhole was full of water all the way to the top. This is a key observation, since only one additional bend and two short lengths of strait pipe separate this point from the RCP discharge invert at 12.36 ft elevation. The calculations include a case to determine how much flow would be required to cause the observed level if the ditch were in good working condition. The answer was that flows exceeding 50,000 gpm, would be required. The resident also reported that the water at the surface was calm, not turbulent. This would lead one to believe that there was an obstruction at the ditch, not that the outflow was this very large value.
2. In order to confirm that the ditch could discharge into its receiving wetland, a backup of water due to "storm surge" had to be eliminated. The data was reported by NOAA in their Hurricane Florence Report. They reported both the maximum Storm Tide and the maximum "Inundation" (water level on normally dry ground) at Snow's Cut, the Cape Fear River, and Southport. The "Inundations" reported were, in all cases, about 2 ft less than the storm surge, and ranged from 3.1-3.5 ft. The storm surge and inundations were relatively low and were well less than the elevation difference to the ditch discharge at 10-11 ft. Note: Snow's Cut, only a few miles to the south on the river, ties the maximum river levels to the nearby sea level.
3. The second receiving wetland question was related to possible back-up due to the presence of River Rd between the wetlands and the Cape Fear River. This issue is negated by the fact that a nearby segment of the River Rd is below 5 ft elevation. Stormwater from the wetland would simply pass over the road and water surface elevations above 5- 6 ft would pass extremely large flows
4. Pictures taken immediately after the storm show the RCP discharge to be significantly obstructed by soil build-up in the ditch. The pictures seem to indicate that the obstruction was worse than the 2 ft elevation of obstruction that I used in Study Cases 3 & 4. The pictures also don't preclude a possible soil build-up in the throat of the "flared end section. The "flared end section" tapers from full round pipe to a bottom only section of double width [3.0' RDP inlet to 6'wide by 1.5' height at discharge]. The combination of open top and obstruction immediately downstream of the "flared end section" would explain the observation during the storm that the water flow was discharging upward.

FIGURE 7: NHC Topographic Map of Receiving Ditch Area



List of Figures, Maps & Tables

This Document prepared by: W. L. Nutt ; BSChE; Research Fellow (Ret), E. I. DuPont

Wayne L Nutt Date: 3/2/202

References:

1: Cameron Hydraulic Data; 17th edition; Ingersoll-Rand Corp.

2: Crane Flow of Fluids, through valves, fittings, and pipe; Technical Paper No 410
Also partial avail as; CraneNuclear-A4-Aprl2013-Section8.pdf

3: [www.Eng.Auburn.Edu>Handbook](http://www.Eng.Auburn.Edu) > Open Channel Flow Calculator
Trapezoidal Ditch; calc flow and depth (Manning Equation)

4: [Perry's] Chemical Engineers' Handbook; Sec 6 - Transport & Storage of Fluids;
Table 5-7 (roughness values), Fig 5-12 (Ave *n* Values), Fig 5-26 (Moody Diagram)

5: Chemical Process Design Standard, E I DuPont deNemours; Friction Loss in Fittings & Valves -DG 5B

6: U S Army Corp of Engineers; Design Manual, Chap 2: Open Channel Hydraulic Theory
Table 6.1, roughness coeff. (p43) Moody Diag., friction factor vs. roughness (p39); Engineering Manual EM 1110-2-1601; Hydraulic Design of Flood Control Channels

7: New Hanover County discovery documents: BCH_Defendants
- Doc 110112: Map of off-site contributing area
- Doc 000110: Tripp Eng, preliminary diverter box design
- Doc 109785: Early problem definition documents & correspondence
- Doc 000091: Ref DWGS
- Doc 022463: TRC Agenda docs show Tralee Ct drainage; see C1 & C2
- Doc 109040: NHC; soil types near Cathay Rd/Lipscomb Dr
- Doc 96606-616: "Kirkwood" (adjoining sub) maps & drainage
- Doc 004744/57: Flooding Pictures

8: Reiss & Nutt plaintiff discovery documents: Plaintiff 1-72 (1).pdf

9: www.Weather.Gov NOOA/National Weather Service/NOOA/ Hurricane Florence Report